The design of crane girders

Recent correspondence in *Verulam*[1] suggested that there were no decent examples of crane girder design to the Eurocodes. David Brown of the SCI rises to the challenge…

The problem

According to the contribution in *Verulam*, a number of problems exist with the design of a mono-symmetric member (a plate welded to the top flange of a UB) and destabilising loads:

- BS 5950 examples have 'mysteriously disappeared' from the equivalent Eurocode publications.
- The only way to design the member is to use 'a piece of software from a French website'.
- There is no way of checking the result (from the French software).
- Gantry girders would have to be doubly symmetric, or have the top flange fully restrained.

What are the options?

Looking back at the BS 5950 examples in the SCI library, most are mono-symmetric with a channel welded to the top flange. An example with a plain plate welded to the top flange is presented in early editions of the 'Red Book'[2].

Some of the examples calculate the section properties of the compound section – not a precise task, (especially before channels had parallel flanges) and verify the fabricated member on that basis. Alternative examples adopt the traditional and simpler approach of assuming that the additional plate (or channel) carries the horizontal loads, and the rolled section carries the vertical loads.

If one held the pessimistic expectation that the Eurocodes always adopt the most complex approach, one might be pleasantly surprised to find that the simple approach is allowed in clause 5.6.2(4) of EN 1993-6, which is the Standard covering the design of crane supporting structures. According to this clause, lateral loads are resisted by the top flange, and vertical loads are resisted by the main beam under the rail. This simple approach will be familiar, and facilitates the use of mono-symmetric sections.

Following this simple approach, torsional moments are resisted by a couple acting horizontally on the top and bottom flange. As an alternative, torsion may be treated rigorously.

Lateral-torsional buckling

Gantry girders are unrestrained, and have lateral loads applied at the top flange level (or above). As the beam buckles, the vertical loads may be eccentric to the shear centre, so there are additional torsions on the section, as indicated in Figure 1. Clause 6.3.2.1 of EN 1993-6 insists (quite properly) that these torsions must be accounted for. The designer again has options, according to clause 6.3.2.3.

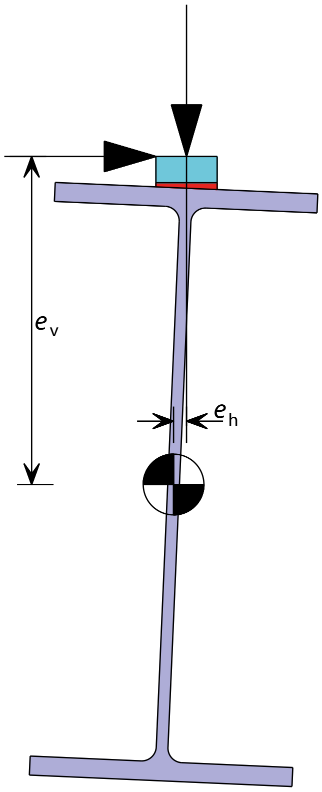

Figure 1 Torsions on a gantry girder

The first option is to simply consider the top flange and part of the web acting entirely alone, and check it as a simple strut. Safe, certainly, but conservative. The second option is to assess the member for the combined effects of lateral-torsional buckling, minor axis moment and torsion, using the interaction expression presented in Annex A of the Standard. The UK National Annex endorses the use of this alternative.

Of course, the interaction expression looks complicated:

$$\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} + \frac{C_{Mz} M_{z,Ed}}{M_{z,Rk}/\gamma_{M1}} + \frac{k_w k_{zw} k_\alpha B_{Ed}}{B_{Rk}/\gamma_{M1}} \leq 1$$

A numerical worked example would help, as the correspondence in *Verulam* notes. Fortunately there is a full worked example in P385[3], which is SCI's publication on the design of steel beams in torsion. Example 2 is precisely the case under consideration – a gantry girder, except the selected member is a UB with no plate. Because this comprehensive numerical example exists, no further attention is paid to the interaction expression in this article.

Destabilising loads

Loads that move with the buckling compression flange are classed as destabilising. As the correspondence in *Verulam* indicates, one would normally assume that gantry girders are subject to destabilising loads.

EN 1993-6 offers an interesting twist (no pun intended) to the classification of destabilising loads. Clause 6.3.2.2 suggests that if the crane rail is fixed directly to the runway beam, the applied vertical load can be considered as stabilising. This unexpected conclusion is because, as shown in Figure 2, as the runway beam starts to twist, the application of load moves to the 'high' side of the rail, which is actually on the 'restoring' side of the shear centre. Thus the load is stabilising and in these circumstances the Standard notes that it may be assumed that the loads are applied at the shear centre. ▶26

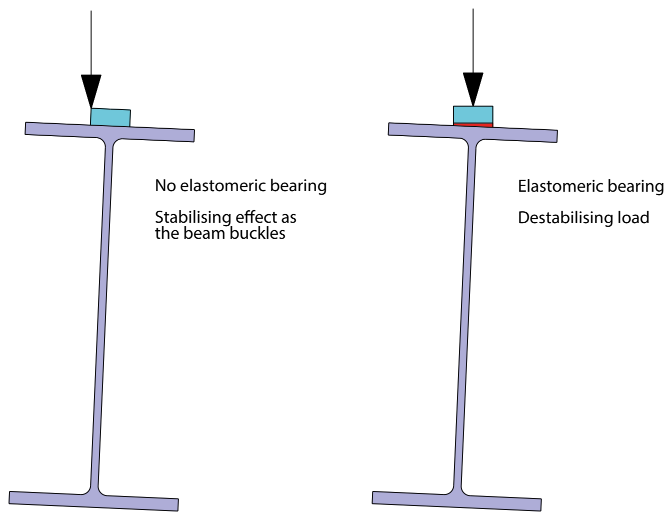

Figure 2 Influence of crane rail on load classification

▶24 If the rail is supported on a flexible elastomeric pad, the loads are destabilising and the Standard notes that the loads should be assumed to be applied at the top of the flange.

In BS 5950, destabilising loads were treated by multiplying the system length by 1.2 (typically), with further adjustment depending on the support conditions. The equivalent uniform moment factor m_{LT} had to be taken as 1.0 (so no benefit from the shape of the bending moment diagram). The Eurocode deals with destabilising loads by adjusting the calculated value of M_{cr}, which will lead us to the comment about using software from a French website.

Calculation of M_{cr}

The background to the problem of M_{cr} is that BS 5950 presents bending strengths p_b for different values of slenderness, λ_{LT}, which is very convenient for the designer, as long as one is not interested how the values have been derived. If interest is sparked, Annex B of BS 5950 provides the background. With patience and algebraic dexterity, one can demonstrate that the BS 5950 terms depend on a familiar friend – the elastic critical buckling moment, M_{cr}. This has been discussed previously[4].

M_{cr} can be calculated using a formula. The version of the formula which allows for destabilising loads is perfectly amenable to computation by paper, pencil and calculator as the *Verulam* correspondence wished. Software solutions merely make the process easier and, many would say, less open to error. After extensive experience asking course delegates to complete a manual calculation of M_{cr} even without destabilising loads, the conclusion is that generally over 80% fail to compute the correct answer. Sadly, the main problem is that delegates attempt to use inconsistent units within the calculation. Maybe software is safer after all.

The French software mentioned is *LTBeam*, which has been discussed several times. Despite the assertion in *Verulam*, independently written software from the UK (does that make it better?) exists and is freely available at *steelconstruction.info*

If necessary, these two programs could be used for mutual checking, and then proved by hand calculation – though a spreadsheet is strongly recommended to remove the tedium of the latter option.

How to check?

The calculation of M_{cr} is merely a step on the way to the result, so checking of the final resistance is probably wise. Options are available, starting with a 'sense check' against the results from BS 5950. Since the introduction of the Eurocodes the consistent message has been that the structural mechanics has not changed, so one would not expect to find significant differences in the results obtained by either code. Generally, the LTB resistance according to the Eurocode is a little higher than according to BS 5950, so that needs to be recognised, as well as taking m_{LT} = 1.0 for destabilising loads.

The wise authors of BS 5950 recognised that increasing the effective length of the member was a good way to allow for destabilising loads. That simple check can be completed by looking at the calculated member resistances for the two lengths.

Simple design assessment

Some straightforward checks of the example presented in P385 have been completed. The example demonstrates the verification of a member subject to combined major and minor axis bending combined with torsion, but if the example is reconfigured to assume lateral loads (and torsional effects due to eccentricity) are taken by a plate welded to the top flange, the exercise becomes a review of the main member.

The vertical loads are destabilising, so according to EN 1993-6 are assumed to be applied at the level of the top flange. Accounting for the position of the loads, M_{cr} = 320 kNm*, according to P385, and M_b = 277 kNm*.

The span of the gantry girder is 7.5 m, so applying a factor of 1.2 results in a span of 9 m. Then one must make a reasonable estimate of the shape of the bending moment diagram, or conservatively assume that C_1 = 1.0

Looking at the bending moment diagram (Figure 3), it looks vaguely similar to that for a UDL, admittedly with some angularity, but for a quick check, assume that $C_1 = 1.13$, mainly for easy use of the look-up tables in the Blue Book.

For the trial section of a 533 × 210 × 101 UB in S275 (note that all beams are S355 nowadays!), a buckling length of 9 m and $C_1 = 1.13$, the buckling resistance $M_b = 288$ kNm. As a coarse check, this is quite reassuring when compared to the computed value of 277 kNm*.

A further approach is to use the look-up tables in the back of P362[5], where χ_{LT} depends only on h/t_f and L/i_z, which more mature designers will recognise as D/t and L/r_{yy} in previous nomenclature. The tables in P362 assume $C_1 = 1.0$, so are likely to deliver a smaller resistance than computed with precision.

$h/t_f = 536.7/17.4 = 31$

$L/i_z = 9000/45.7 = 196$

Using Table E2 from P362, $\chi_{LT} = 0.38$ with some approximate interpolation.

Therefore $M_b = 0.38 \times 2610 \times 10^3 \times 265 \times 10^{-6} = 262$ kNm

This seems to offer reassurance that we are in the correct parish, at least, when compared to the computed value of 277 kNm*.

What has not been addressed!

In the opinion of the author, the challenge with gantry girders is not in fact the member verification, but the determination of the applied actions in accordance with EN 1991-3, a problem which was not mentioned in *Verulam*. A treatise on the subject is available for download[6], but the topic is complex.

Other issues not addressed here are the deflection limits for crane supporting structures, which may be more important than the member resistance. Designing the supporting structure to control the spread of the gantry beams will be important. Finally, fatigue design may govern the size of the member – an introduction to the subject[7] and example calculations[8] have been published in NSC.

*Footnote

Readers trying to replicate the calculation of M_{cr} as quoted in P385 may have some difficulty. The correct value of M_{cr} appears to be between 336 and 340 kNm and consequently $M_b = 288$ kNm. Although it would be tempting to blame the software, it appears the user calculated the level of load application as 533/2 + 65 = 331 mm, when 286 mm should have been used (the load is applied at the top flange, not on top of the 65 mm rail).

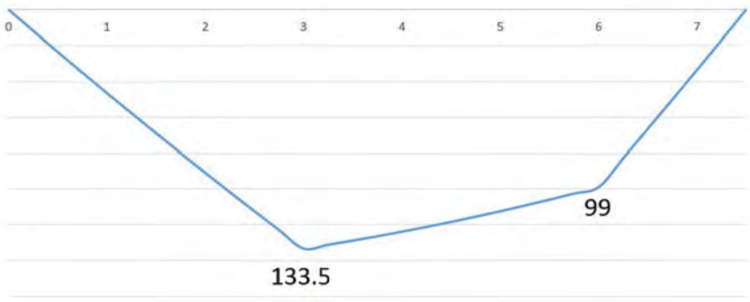

Figure 3 Bending moment diagram

References

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7 Henderson, R. Introduction to fatigue design to BS EN 1993-1-9. New Steel Construction, September 2018
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